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Le-Wu Lu

Shiunn-Jang Wang

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SEISMIC DAMAGE ANALYSIS OF THE MORISADA BUILDING

Le-Wu Lu[1] and Shiunn-Jang Wang[2]

Introduction

On June 12, 1978 a strong earthquake with a magnitude of 7.4 on the Richter scale shook a major part of the Miyagi Prefecture in Japan. Hardest hit was the city of Sendai. There was a substantial loss of life and many of the city's structures, some quite modern, suffered severe damages. The characteristics of the earthquake and its damaging effect are described in a reconnaissance report published by the Earthquake Engineering Research Institute (1) and in an investigative report by the Architectural Institute of Japan (2). With the assistance of Professor S. Kuranishi of the Tohoku University, the senior author was able to spend two days visiting that city and inspected the damages to various buildings. One of the severely damaged buildings is a four-story steel structure which housed the Morisada Kosyo Company.

Description of the Building

The building, completed in 1971, was 13.3m long, 9.8m wide and 12.3m high, excluding the roof tower. Figure 1 shows the general layout and the dimensions of the structure. It had three bays in the N-S direction and one bay in the E-W direction.

In the N-S direction, the structural system consisted of two identical moment-resisting frames, spaced at 7.3m apart. The floor slab extended 0.85m and 1.65m in the E-W direction beyond the center-lines of these frames. The structural system in the E-W direction consisted of four moment-resisting frames with overhangs.

There were two girder sizes in the three-bay frames, G1 for floor levels 1 and 2, and G2 for levels 3 and 4. The girders in the one-bay frames were G3 for levels 1 and 2, and G4 for levels 3 and 4. Three column sizes were used in the building, C1 for the first story, C2 for the second story, and C3 for the third and fourth stories. All the members, except C1 were standard Japanese rolled H shapes. As shown in Fig. 1, all the columns in the three-bay frames were oriented in their weak direction. To increase its strength and stiffness, the C1 column had two 9mm thick plates welded across the flange tips to form a box section. The member sizes of the structure are given in Table 1.

[1]Professor of Civil Engineering, Fritz Engineering Lab, Lehigh University, Bethlehem, Pa. 18015

[2]Research Assistant, Fritz Engineering Lab, Lehigh University, Bethlehem, Pa. 18015

The structural design of the frames was based on the weak-column, strong-girder concept, which is not generally adopted by the U.S. engineers.

Observed Damage

The major damage observed after the earthquake was a very large permanent deflection in the N-S direction between floor levels 2 and 3. This is illustrated in Fig. 2. The permanent deflection amounted to about 1/20 of the story height. Also a relative displacement of 13cm was measured over the clear height of one of the C3 columns (Fig. 3). The column yielded extensively at the two ends, where flange local buckling was also observed. The excessive distortion led to shear failure of the exterior walls and cracking of the window glasses and partitions in the third floor. Some cracks were also found around the column footings.

Previous Studies

There are several reports in Japanese containing information about this building. A check of design adequacy of this building has been carried out in Ref. 3. The design was found to satisfy all the requirements of the applicable Japanese code. Also, the quality of construction of this building has been investigated and found to be very satisfactory (4). Furthermore, dynamic analyses with elastic material properties, shear building and lumped mass assumptions have been performed (2,3,4). The energy method developed by Kato and Akiyama (5) has been used in accessing the safety of the structure. In this method, the safety of a structure is evaluated by comparing a structure's energy dissipating capacity with earthquake input energy. The results show that the energy concentration ratio is the highest between the second and third floor levels. It is therefore concluded that the observed damage in the columns was due to high energy dissipation required of them. No report, however, offers any clear explanation why the energy was so highly concentrated between these floors. Another suggestion is the sudden change of the column stiffness at the second floor level that caused the problem(3). But, no study has been carried out to support this suggestion.

Seismic Analysis

Time-history analyses of the building have been performed using the accelerations recorded on the first floor of the Architecture and Civil Engineering Building of the Tuhoku University. The university is located on a hill, 7km away. The N-S component of the record with a peak acceleration of 258 gals. (Fig. 4) has been used in the analysis. The response spectrum of the record is given in Fig. 5, which shows a peak at about one second, very close to the natural period of the structure.

The DRAIN-2D program is used to perform most of the calculations. The concepts and features of the program are described in Ref. 6. The structure is idealized as a planar assemblage of discrete elements and the analysis is done by the Direct Stiffness Method with the nodal displacements as the unknowns. First-order analysis, in which the effect of P-Δ moment is ignored, has been performed for a mass-proportional damping equal to 5% of the critical camping of the first mode. Different strain-hardening properties have been selected and incorporated into the analysis. Figure 6 shows the roof deflection vs. time relationship of the building for a strain-hardening modulus of 0.0074E (E = Young's modulus) and allowing a 21% increase of the plastic moment of the columns. These values were recommended by Kabe and Rea based on some shaking table tests of small-scale steel frames (7). The maximum roof deflection has been found to be 20.4cm with a permanent deflection of 9.3cm. The latter figure is to be compared with the 13cm column deflection mentioned before. A later analysis using a computer program developed by Professor F. Y. Cheng at the University of Missouri-Rolla and taking into account the P-Δ effect shows a considerably larger permanent deflection. This indicates that a significant portion of the permanent deflection may be due to the P-Δ effect.

The research also includes a study of the ways to improve seismic response of the building. Three different options have been considered: the first is to retain the moment-resisting frame scheme, the second is to add X braces in selected bays and stories, and the third is to install eccentric or concentric K braces. In these studies, no consideration has been given to the possible architectural or functional restrictions imposed by the building user. In this paper, only the results of the study of the first option are presented. The results of the other options can be found in Ref. 8.

One way to improve the structural response of the moment-resisting frame is to increase its column sizes. The idea is to force more plastic hinges to form in the girders and to distribute the plastic deformation more uniformly throughout the structure (instead of concentrating in a particular story). Four schemes of increasing column sizes have been investigated; they are shown in Fig. 7. The original three-bay frame of the Morisada building is shown at the center. In System 2 the C1 columns are extended to the second story and the column schedule becomes C1, C1, C3, C3. In this case, the third story remains to be the weakest story. In System 3, the C2 columns are extended to the third story, making the second story to be the most critical story. The column schedule is C1, C2, C2, C3. System 4 is a combination of Systems 2 and 3, with the following column schedule: C1, C1, C2, C3. The weakest story is again the third story, but it should be stronger than the third story of the original structure and of System 2.

In System 5, the C1 column is used for all the stories. The results of the study are summarized in Fig. 8, in which the floor level deflections at the maximum response are given. The original frame shows a substantial increase of lateral deflection in the third story. This is consistent with the observed large permanent deflection in that story. For System 2, the deflection profile is only slightly changed and structural damage may again be concentrated in the third story. The behavior of System 3 is improved somewhat (less roof deflection) with perhaps smaller permanent deflection occurring in the second story. System 5 has the best performance with almost all the plastic hinges forming in the girders. It is a strong-column, weak-girder structure. The locations of the plastic hinges in the five structures are shown in Fig. 9.

Summary and Conclusions

An analytical investigation of the seismic response of the Morisada building which was damaged severely during the June, 1978 Miyagi-Ken-Oki earthquake has been carried out. Time-history analyses of the damaged moment-resisting frame have been performed using a ground motion record from a strong motion instrument located 7km away. Similar analyses have also been performed on the frame with increased column sizes.

The major conclusions of the study are:

1. The observed large permanent deflection in the third story is due primarily to the high flexibility of the structure and the sudden change of story stiffness.
2. The inelastic seismic analyses predicts very well the damage pattern. The calculated permanent sway is in fair agreement with the observed value, considering the uncertain nature of the ground motion actually experienced by the building.
3. The dynamic characteristics of the structure may have been significantly affected by overall instability (P-Δ effect).
4. Moment-resisting frames designed by the weak-column, strong-girder concept may experience large inter-story sway during a strong earthquake.

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Table 1 Member Sizes of the Building

Type of Member	Designation	Size
Girder	G1	H-294 x 200 x 8 x 12
Girder	G2	H-244 x 175 x 7 x 11
Girder	G3	H-336 x 249 x 8 x 12
Girder	G4	H-294 x 200 x 8 x 12
Column	C1	H-300 x 300 x 10 x 15*
Column	C2	H-300 x 300 x 10 x 15
Column	C3	H-250 x 250 x 9 x 14

*(plus 2-9mm plates)

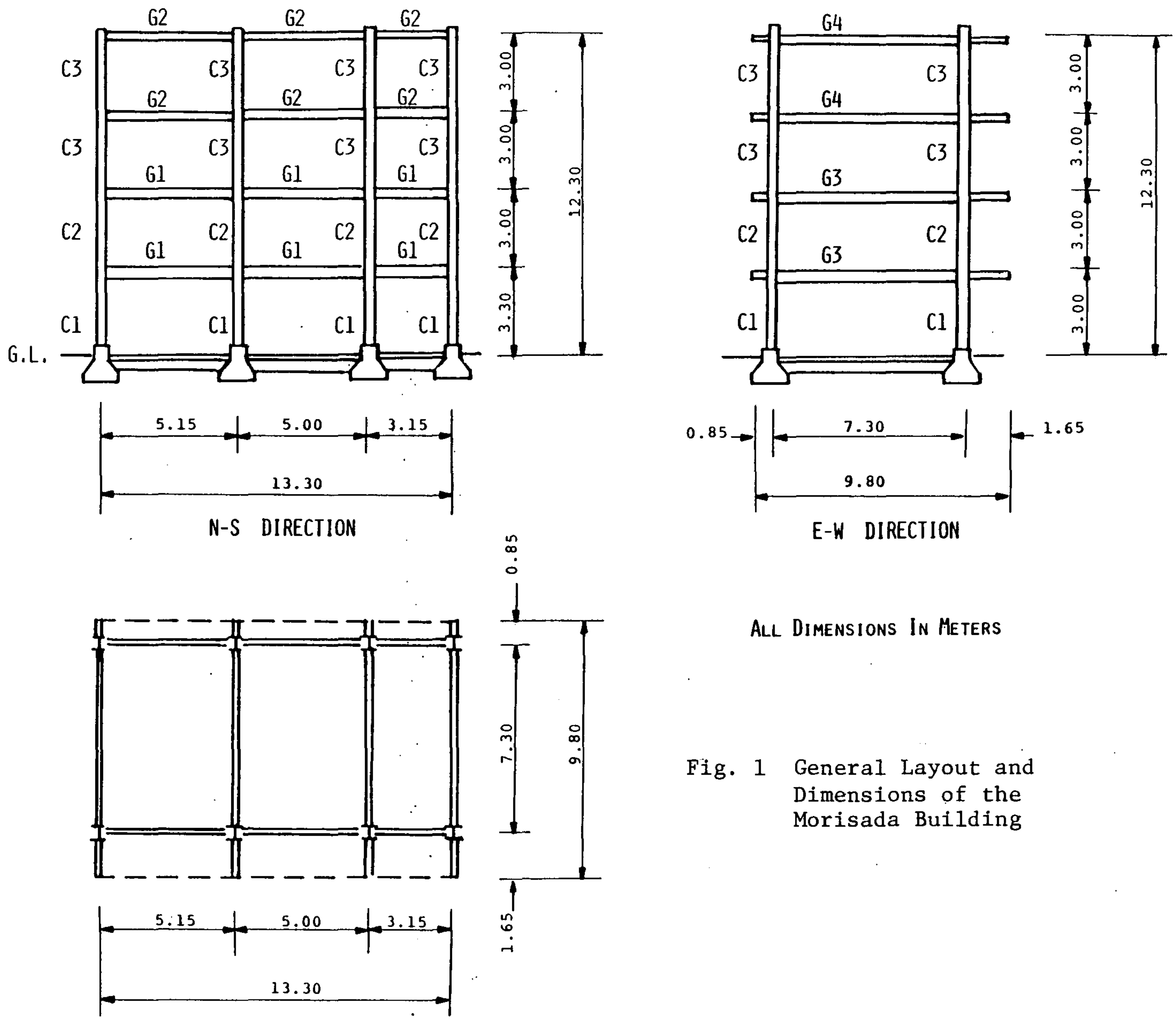

Fig. 1 General Layout and Dimensions of the Morisada Building

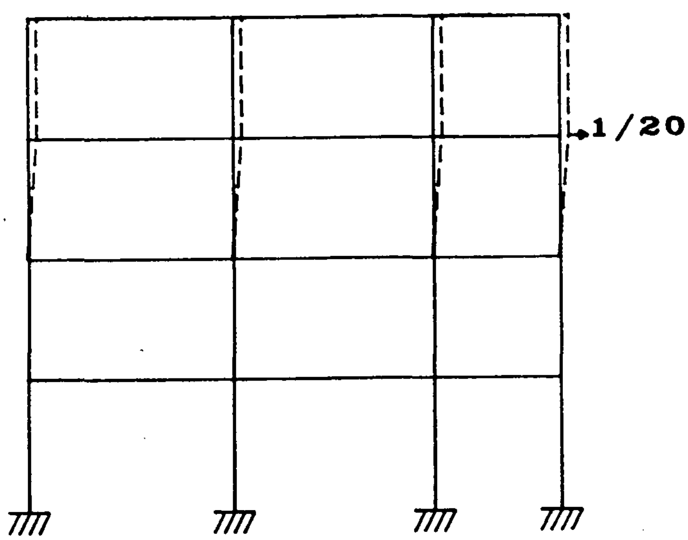

Fig. 2 Observed Permanent Sway

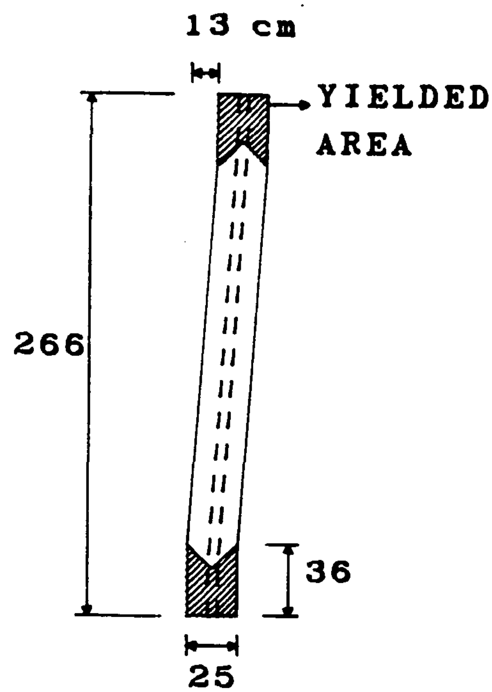

Fig. 3 Damaged Column in Third Story

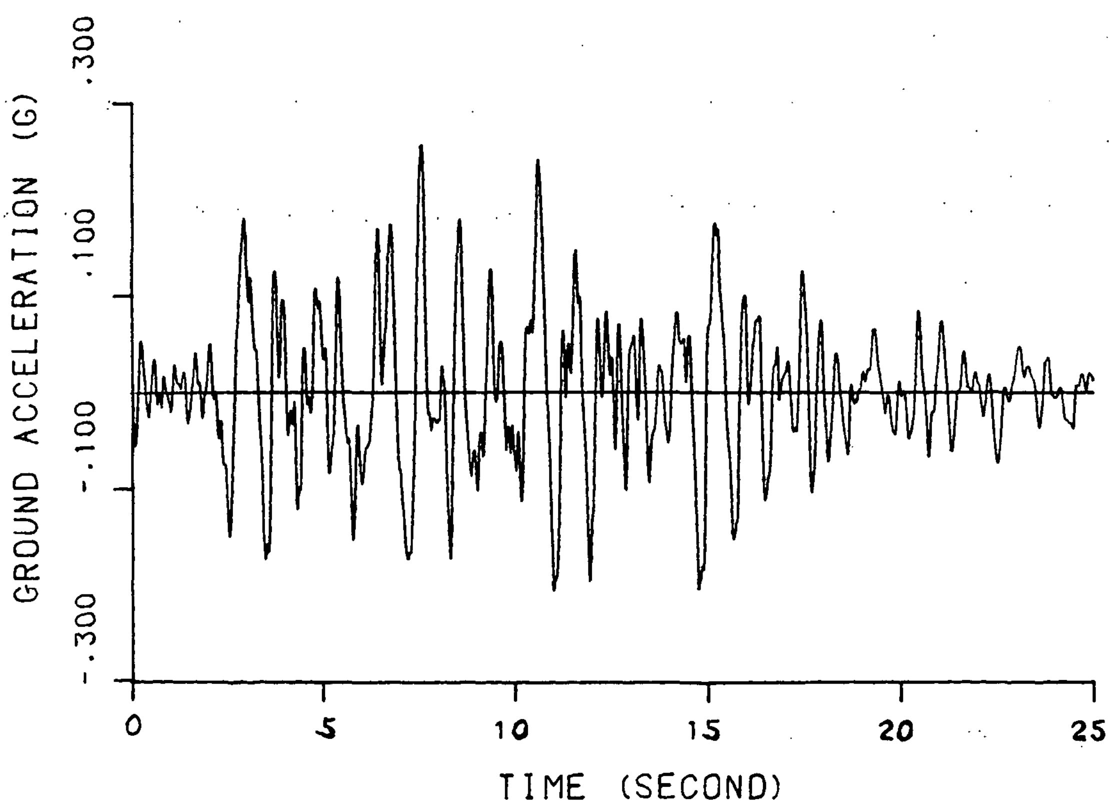

Fig. 4 Ground Motion Record Used in Study

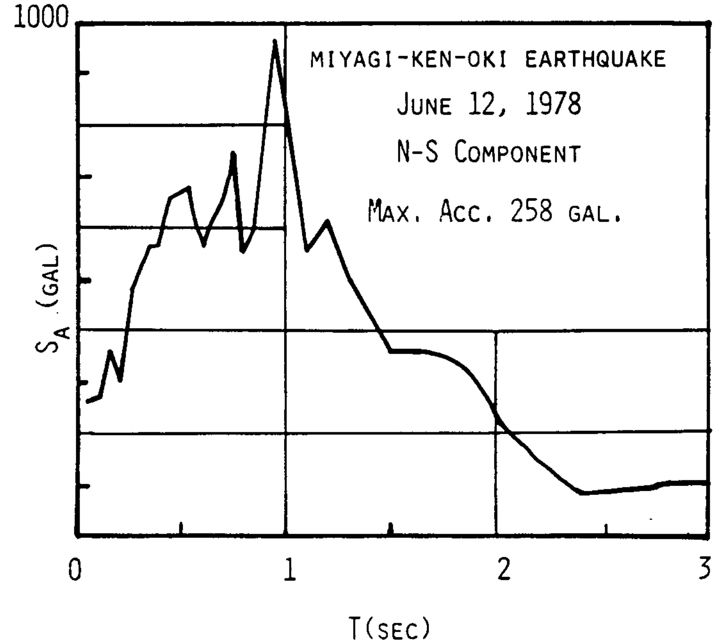

Fig. 5 Response Spectrum of Ground Motion Record

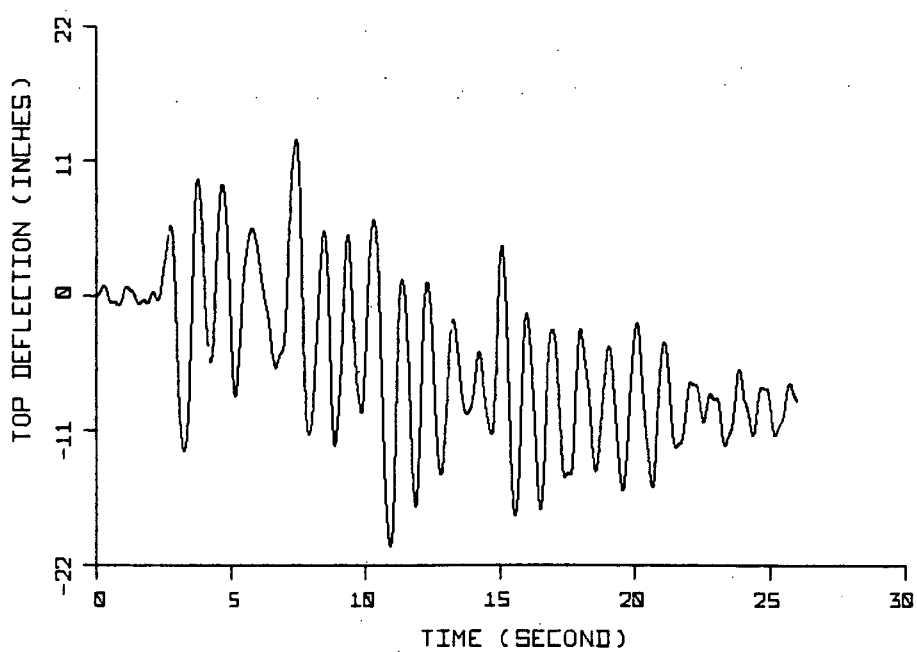

Fig. 6 Response History of Roof Deflection

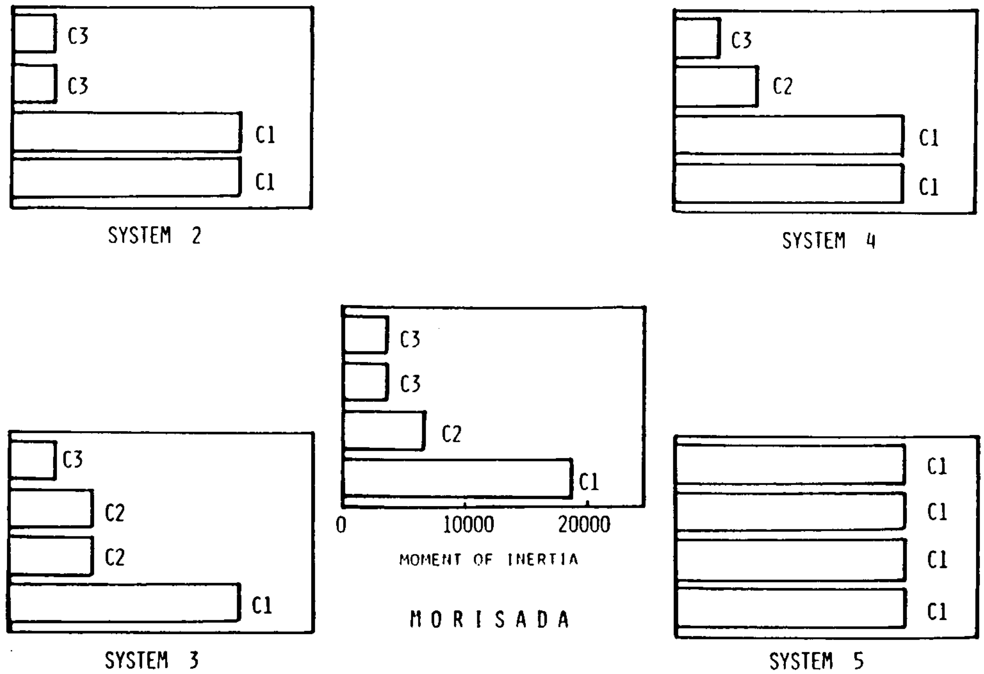

Fig. 7 Column Sizes in Modified Morisada Building Frames

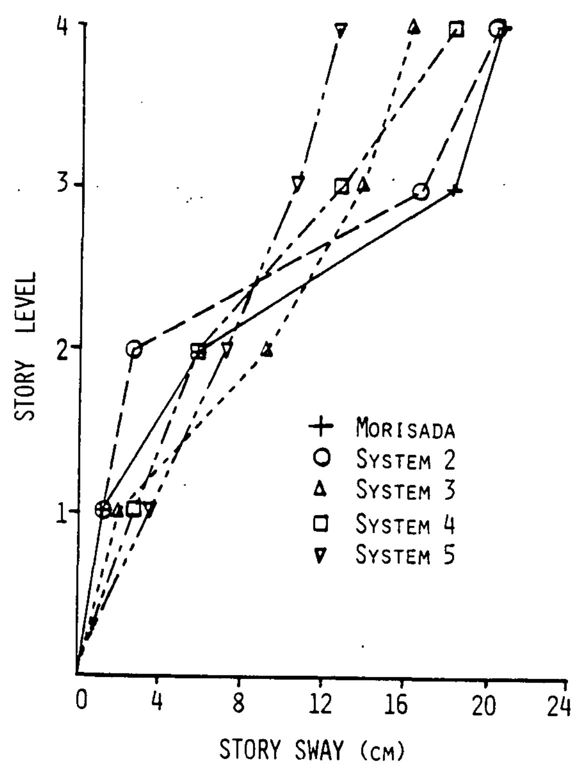

Fig. 8 Story Deflections at
Maximum Response

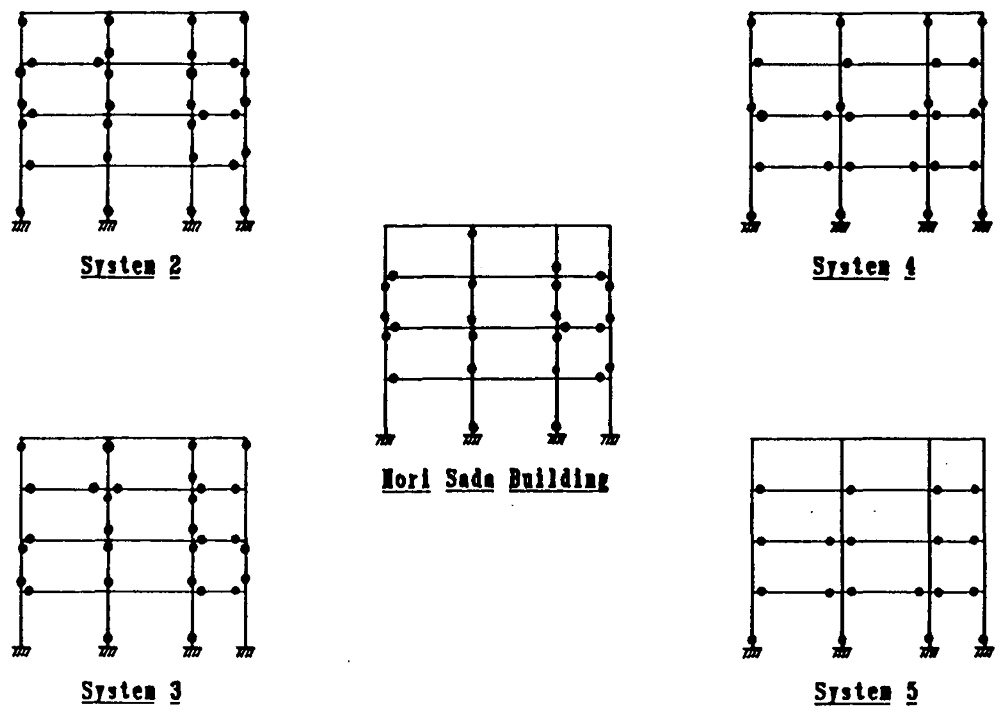

Fig. 9 Plastic Hinge Locations